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Journal of the
POWER DIVISION
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HIGH-LIFT CONSTRUCTION METHODS FOR MASS CONCRETE

Otto Holden,[1] M. ASCE

SYNOPSIS

The high-lift method of concrete placement has been applied successfully by Ontario Hydro since 1930 to a series of structures ranging in height to a maximum of 240 feet. All evidence from crack surveys, embedded instruments and stress analyses indicates that dangerous cracking is unlikely in these structures.

INTRODUCTION

The high-lift method of concrete construction as practiced by Ontario Hydro was first adopted within the organization in 1930 at the Chats Falls Power Development on the Ottawa River. In the relatively long, low structure it was found practicable to place concrete continuously in each bulkhead section from the foundation to the full height of the dam, a maximum of 60 feet.(1) This method of placing was found to increase the rate of construction, to eliminate troublesome horizontal construction joints and to facilitate winter concreting. In spite of the seemingly severe combination of high internal temperature at the time of form removal coupled with low winter temperatures there has been no evidence of internal cracking from this or any other source.

Following the successful completion of Chats Falls Power Development the high-lift system was used for a number of small structures for which it proved highly suitable. In 1939, when Barrett Chute Dam, approximately 100 feet in height, was being designed the alternative methods of construction were given serious consideration. In conjunction with this review theoretical studies related to mass concrete behaviour were undertaken. These were directed mainly towards revealing the influence of thickness of lift on the temperature

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1. Chf. Engr., The Hydro-Electric Power Comm. of Ontario, Toronto, Canada.

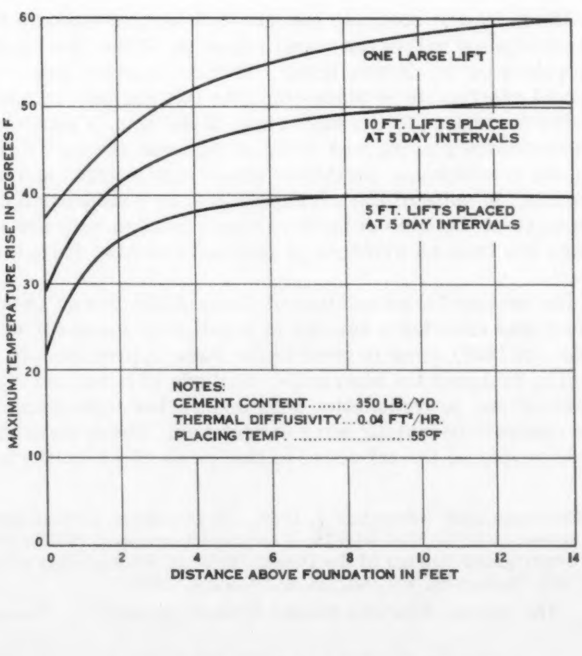
rise and subsequent cooling stresses.

Calculations were made of the maximum temperature to occur in mass concrete for a variety of placing procedures including one single lift of great thickness, and the alternatives, successive lifts of 5 and 10-foot thickness placed at various intervals. For the conditions assumed it was found that the peak temperature could be reduced about one-third below the adiabatic rise if 5-foot lifts were placed at intervals of 5 days, and about one-eighth if 10-foot lifts were placed at the same interval, Figure 1. If the height of lifts exceeded 15 feet, the temperature rise at the centre was found to be essentially adiabatic.

Estimates of the tensile stress in the concrete, near the base, due to the temperature rise and fall, indicated the stress at this location varied essentially with the peak temperature. However, the internal stresses in the mass concrete at some distance from the base, which were the result of temperature differentials, were not appreciably different for the low or high-lift placement.

The surface stresses in the concrete for either system of placement were estimated to be chiefly dependent on the "thermal shock" rather than the peak temperature of the concrete. Consequently, efforts were made to reduce thermal shock in winter by retaining the side forms and the top cover on a lift for as long a period as possible after concrete placement. This procedure was feasible owing to the extensive use of local timber for form construction.

From the foregoing studies it was concluded that if a crack should occur near the base of a monolith placed by the high-lift method the crack would probably be restricted to the lower portion of the first lift. Similarly, it seemed unlikely that surface cracks would extend a significant distance into



MAXIMUM TEMPERATURE RISE IN MASS CONCRETE FIG. 1

the interior. Hence, as high lifts were favoured for a number of reasons, including wartime expediency, the system was adopted in the construction of Barrett Chute Dam. Although the forms were built in one stage to the full height of the dam the deepest sections were placed in two lifts. In subsequent structures the forms were erected to accommodate a single lift, usually ranging in height between 40 and 50 feet. During the period from 1940 to 1950 when progressively taller structures were being designed, serious consideration was given at each stage to the suitability and safety of the high-lift system of construction. However, observations of the performance of existing structures indicated that the construction practice had produced sound concrete and hence supported the conclusions drawn from theoretical studies. Accordingly, confidence was established in the continued use of the method.

Although the application of the high-lift system to structures ranging in height up to 240 feet may have been the outcome of circumstances peculiar to Ontario Hydro, the lack of internal cracking that seems inherent in these structures in spite of an unusually great cooling range may be of considerable interest. Consequently, this paper will deal with those features of design, construction and performance that appear to have a bearing on the nature and limited extent of cracking that has been experienced by this organization.

Factors Favouring High-Lift Practice

As with most other organizations, the construction practices followed by Ontario Hydro have been the outcome of a large number of influencing conditions in which compromise to speed and efficiency has played an essential part. Growth of electrical load since 1940 has continued at an unprecedented rate; consequently a strong emphasis has been placed on rapid completion through closely integrated activity on all phases of a project. Factors that have played an important part in extending the practice adopted at Chats Falls to later structures have been the range in size of structures, climatic conditions, nature of cement and aggregate supplies, equipment shortages and a desire to eliminate horizontal joints.

Size of Structures

In order to avoid possible misconceptions as to the field in which the high-lift method has been applied most extensively, the following outline is given of Ontario Hydro activities in the development of additional hydraulic power since the beginning of World War II. As indicated in Table 1, the majority of the power and storage dams are less than 200 feet in height and many do not exceed 100 feet. Concrete quantities for the various projects, including all auxiliary structures, fall generally in the range of 60,000 to 450,000 yards. In addition to the main structures, these power developments have included many auxiliary dams, powerhouses, control works and plant extensions usually involving smaller yardages than the main structures. However, these have contributed appreciably to the Commission's total quantity of mass concrete in service. In such auxiliary structures, particularly, the possibility of achieving a saving in cement through the adoption of low-slump large-aggregate concrete was obviously much less than it would have been in a single major structure of equal yardage. The use of plastic concrete throughout an entire project has permitted great flexibility in the selection and use of handling methods, including belt conveyors, dump and ready-mix trucks, concrete pumps and buckets.

RECENT MAJOR HYDRAULIC DEVELOPMENTS OF
THE HYDRO-ELECTRIC POWER COMMISSION OF ONTARIO

NAMES OF DAMS AND GENERATING STATIONS	LOCATION	IN SERVICE DATE	MAXIMUM HEIGHT		CONCRETE YARDAGE		
			DAM [FT]	LIFT [FT]	MAX. LIFT	STRUCTURE	PROJECT
CHATS FALLS	OTTAWA RIVER	1931	60	60	1,100	160,000	254,649
OGOKI DIVERSION [2 DAMS]	OGOKI RIVER	1942	56	51	1,280	42,400	49,620
BARRETT CHUTE	MADAWASKA RIVER	1942	97	76	1,430	62,000	87,193
STEWARTVILLE	MADAWASKA RIVER	1948	215	58	11, 790	244,883	278,778
AGUASABON	LAKE SUPERIOR	1948	120	48	3,780	82,559	116,930
PINE PORTAGE	NIPIGON RIVER	1950	140	50	5,240	346,492	378,376
GEORGE W. RAYNER	MISSISSAGI RIVER	1950	235	64	12,350	195, 220	213,857
DES JOACHIMS MAIN DAM	OTTAWA RIVER	1950	190	67	3,500	401, 041	589,869
DES JOACHIMS CONTROL DAM	MC CONNELL LAKE	1950	130	55	6,460	272,638	273,232
CHENAUX [3 DAMS]	OTTAWA RIVER	1951	75	54	1,660	105, 820	238,296
POWERHOUSE	OTTAWA RIVER	1951	—	45	2,885	119,600	
OTTO HOLDEN	OTTAWA RIVER	1952	140	53	3,880	249,252	420,806
SIR ADAM BECK—NIAGARA NO. 2	NIAGARA RIVER	1954—1958					2,076,140
INTAKES, CONTROL DAMS AND REMEDIAL WORKS		1954				281,640	
TUNNELS NO. 1 AND NO. 2		1954				1,182,661	
POWERHOUSE AND HEADWORKS		1954—1958				411,529	
PUMPING GENERATING STATION		1957				188,330	
MANITOU FALLS	ENGLISH RIVER	1958	80	33	670	8, 759	47,028
WHITEDOG FALLS	WINNIPEG RIVER	1958	75	34	1,610	37,165	83,889
ROBERT H. SAUNDERS	ST. LAWRENCE RIVER	1958	156	43	3,860	350,000 *	994,000
CARIBOU FALLS	ENGLISH RIVER	1958	75	51	1,730	57,905	105,508

* MASS CONCRETE PORTION OF POWERHOUSE

Climatic Conditions

In order to meet difficult completion deadlines and to minimize dislocation of construction forces it has been necessary for Ontario Hydro to continue the concreting of most structures throughout the winter months. The requirements for cold weather protection in three representative locations in the Province of Ontario are shown in Figure 2. It can be seen that to stop construction to avoid winter conditions would seriously interfere with progress and would introduce a further element of uncertainty in construction schedules. Experience has demonstrated winter concreting to be entirely feasible. If adequate precautions are taken satisfactory structures will result.(2)

It has been found that with "vertical" construction the area of horizontal concrete surface, and hence the cost of providing protection, is kept to a minimum. The high forms serve as sidewalls for the heated enclosure and as supports for the roof system. Hence, the high-lift system has facilitated winter concreting.

Cements and Aggregates

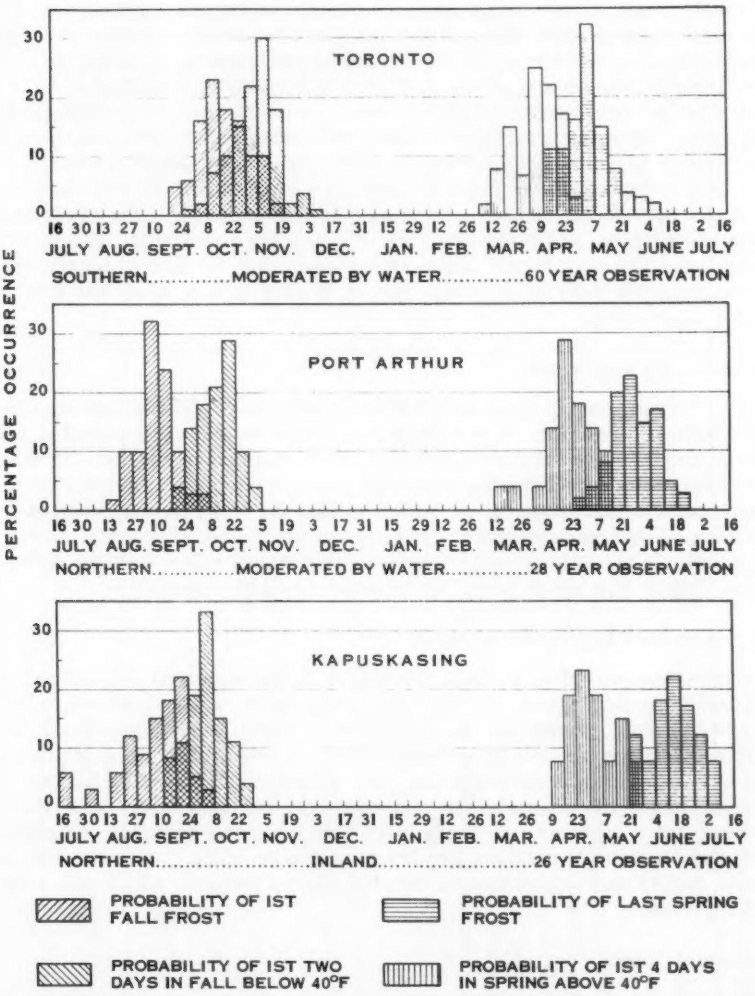
The lack of low heat and modified cements has discouraged a departure from high-lift practice. It has been recognized that the full potential reduction in temperature rise could not be achieved efficiently in low-lift construction when Type I Portland cement must be used. Another factor that discouraged a transition to low lifts was that the full potential cement economy of that system was not possible owing to the lack of the cobble size of gravel. In the Ottawa valley, particularly, there was a definite deficiency in the cobble size which could not have been readily overcome.

Equipment and Material Scarcities

During the period of wartime shortages it was necessary to utilize, as far as possible, existing construction equipment or to be faced with serious delays in delivery. From this standpoint there was a tendency to continue existing practice, including delivery by belt conveyor and pump which do not lend themselves to large-aggregate, low-slump concrete. The lumber requirements for the construction of deep forms did not present a serious obstacle in many instances as there were usually ample supplies of timber from headpond clearing operations and from local saw mills. After the war the timber supply was augmented by surplus Bailey bridging which was adapted to many uses.

Horizontal Joints

The great reduction in the number of horizontal construction joints, and consequently the decreased cost of curing and cleanup, resulting from high lifts is an important economic factor. For the smaller structures, at least, it has been estimated that this economy, together with the elimination of costly temperature control measures that might otherwise have been necessary, fully offset the cost of extra cement for plastic concrete. Moreover, in the limited placing areas required by winter concreting or by site restrictions, more frequent horizontal joints would have tended to retard placing operations.



PROBABLE START AND END OF A.C.I. WINTER REQUIREMENTS
TYPICAL ONTARIO AREAS

FIG. 2

Factors in High-Lift Construction Influencing Thermal Behaviour

High-lift construction as practiced in the past has involved the construction of formwork fifty feet or more in height with a close spacing of internal tie rods. The close spacing of tie rods prevents the delivery of concrete by bucket to the point of placement. Short chutes and elephant trunks are therefore necessary in the forms and delivery systems that permit a continuous flow of concrete through the chutes have been favoured. By close inspection, the use of baffle boxes at transfer points, and the provision of concrete of a high degree of workability, segregation effects have been effectively minimized.

In general, exterior concrete has been designed for a water-cement ratio below 0.60 with a minimum compressive strength of 3000 psi. The strength of interior concrete has been 2000 psi or higher. Using sand contents of up to 40 per cent of the total aggregate, a maximum coarse aggregate size of 2 to 3 inches and a slump of approximately 4 inches, as required by the methods of handling and depositing, cement contents have ranged from 460 to 525 lb per cubic yard for exterior concrete and from 330 to 385 lb per cubic yard for interior mass concrete. The cement used has consistently corresponded to a fairly coarsely-ground Type I.

No special precautions to minimize temperature rise in the concrete have been found necessary so that cement replacements, strict limitations on placing temperature or embedded cooling coils have not been employed. When the height of lift exceeds 15 feet, temperature rise characteristics of the interior are essentially adiabatic. With the mixes used in past high-lift construction, the temperature rise in the interior has ranged between 60F and 70F. Similar temperature rises are commonly observed in the exterior zone where greater heat evolution of the rich face concrete is offset by more rapid heat losses. These would obviously represent highly critical thermal conditions by most standards.

While summer ambient temperatures may be high for short periods, the mixing water is usually cool and although placing temperatures as high as 75 degrees Fahrenheit have been recorded, the average is usually below 60F.

Materials—Handling and Processing

a) Cement

It is customary on most projects to handle cement in bulk. Among those listed in Table 1 the only notable exception was the Ogoki Diversion project, north of Lake Superior, where supplies could be moved only after the fall freeze-up. Accordingly, because of protracted storage at both railhead and at construction sites, the cement was shipped partly in steel drums and partly in export-type paper sacks. Normally, however, each major installation requires bulk cement storage and handling facilities. Adequate bulk cement in quantities of 2000 barrels and upwards is stored in cement silos located adjacent to the mixing plants at the various projects. Cement handling to and from the storage silos to the mixing plants is performed by the conventional bucket elevators and screw conveyors or air discharge systems. Depending upon the location of the mixing plant, cement is transported either by railway or highway.

For example, at one mixing plant of the Sir Adam Beck-Niagara Project, bulk cement was received in railway hopper cars and conveyed by a screw conveyor and a bucket elevator to the mixing plant storage silos. In other locations, cement is transported by bulk cement trailers from cement storage silos installed at railway sidings to mixing plant storage silos. In contrast with these methods, cement was blown to storage silos at the Otto Holden and St. Lawrence projects.

b) Aggregate

In Ontario, there is usually an abundance of concrete aggregate, hence local aggregates are normally utilized. During the exploration stages of a project, a program of aggregate investigation is undertaken. All local sources of granular deposits, potential rock quarries and rock to be excavated in the course of construction are investigated and samples are tested to determine whether they are acceptable for the production of quality concrete. Upon completion of studies to determine the most economical aggregate which is acceptable for concrete production, the choice between gravel and sand deposits and quarried rock is usually reached. This choice is not final, however, until process studies are undertaken to select the most economical method of production. Although crushed stone has been used in a number of locations in preference to local gravels, the St. Lawrence project is the only one to date where it has been found desirable to produce a manufactured sand in place of blended natural sands.

Quarry and pit operations and the production and processing of aggregates vary widely depending on the magnitude of the project, the characteristics of the materials and the terrain. Equipment and plant arrangements are selected or designed to suit the conditions found at each site and therefore cover a wide range in complexity depending on local requirements. An essential feature of most handling systems for crushed stone is a re-screening plant immediately preceding the concrete batching plant. This has been found necessary to remove dust coatings and to eliminate fines that accumulate in coarse aggregate stockpiles.

Concrete Production and Delivery

a) Production

Since the concrete quantities and rates of production at the various projects vary widely, the mixing plant installations show a wide diversity in output, size and degree of automation. The plants have ranged from those having one 2-cubic yard mixer producing 40 cubic yards per hour to those having four 2-cubic yard mixers or two 4-cubic yard mixers producing 160 cubic yards or more per hour. For instance, the mixing plant at Manitou Falls GS had one 2-cubic yard mixer, and aggregate and cement batching were performed by manual control. In contrast, the mixing plants for major projects have usually been supplied with fully automatic controls and with up to four 2-cubic yard mixers for concrete production. At the Sir Adam Beck-Niagara project two such plants were required, while at Des Joachims there were three mixing plants with two, three and four 2-cubic yard mixers respectively.

b) Delivery Methods

Final deposition of the concrete in the forms has in most instances been accomplished through elephant-trunk drop chutes; however, there has been a

wide variety of methods used for transferring concrete from the mixing plant to the forms. Depending primarily on the proximity of mixing plant and the structure, methods have ranged from belt conveyors to buckets on flat-bed trucks or trains, dump trucks, transit-mix trucks and concrete pumps. All of the above systems have proved satisfactory under suitable circumstances, although pumps have been preferred for controlled delivery in powerhouse, tunnel and in control works structures. At Chenaux project a total of 238,000 cubic yards was placed by pump in the main dam, powerhouse, control dam and auxiliary structures.

It will be noted that continuity of placing is a feature of the high-lift system. At Stewartville and Des Joachims, for example, concreting proceeded steadily for a week or more at a time in placing each of the lower lifts which ranged in size from 10,000 to 12,000 cubic yards.

Formwork

Although steel forms have been used on a number of occasions, the winter construction program has tended to create a preference for wooden forms with their insulating value and adaptability. The general availability of timber has been a contributing factor in the use of wooden forms.

Design criteria for forms which have resulted from many years of experience generally call for 3/4-inch lumber or plywood sheathing with 2- by 6-inch studs at 16 to 18-inch centers. Wales are made of double 2- by 6-inch material spaced 27 to 30 inches. Strongbacks made of 4- by 6-inch timber are spaced from 48 to 72 inches. Half-inch mild steel tie rods with reusable threaded clamps are generally spaced at 27-inch centers in both directions.

At the George W. Rayner and McConnell Lake Dams, Bailey form cages were used extensively. In this system, which utilized Bailey bridging with numerous modifications, the plywood form panels were 4- by 10-feet in size with double 2- by 4-inch studs instead of the single 2- by 6-inch studs that are normal with the 4- by 8-foot panels used with timber form framing. The Bailey system used steel transoms spaced at 5-foot intervals instead of wales and Bailey trusses at 17-foot centers as strongbacks. Although the system may be used without closely-spaced internal ties they have been found desirable to overcome unsightly deflections.

Cold Weather Concreting

As noted previously, winter concreting is almost a "must" on jobs of any size in Ontario, because of climatic conditions. Records for the province, except for its southern part, indicate that the average mean temperature is less than 32F for six months of the year. For three of these months it is under 10F, and for the months of January and February it is below zero. To stop construction in winter would mean a shutdown each year for several months, a procedure estimated to be more costly than working continuously. Therefore, work goes on regardless of temperature.

With a good stockpiling arrangement and heated reclaiming tunnels, reclaiming conveyors and mixing plant, it may not be necessary to preheat the aggregate unless ice and frost cause jams in the reclaiming chutes. If heating becomes necessary, the best results have been obtained by using steam coils or steam jets. Steam jets are very useful for taking the frost out of sand but their use should be avoided with coarse aggregate because they tend

to freeze the stone into a solid mass. For this reason heated coils will produce the best results, but generally only the sand is heated.

When the aggregate is being stockpiled, care should be taken to ensure a working depth of the piles over the reclaiming tunnel and surrounding ground, otherwise, a sudden frost will penetrate right to the bottom of the pile. This will mean that much more heat will be required to thaw and keep the aggregate moving through the reclaiming tunnel chutes. It has been found that a minimum depth for a pile should be at least 14 feet over the reclaiming tunnel with 60 or 70 feet preferred. On most jobs aggregate is moved from the stockpile to the mixer by conveyor. These conveyors are preferably enclosed and heated for efficient operation. Thus, the convenient and efficient way to carry steam mains to stockpiles and forms is along or inside such a conveyor housing.

The mixing temperature can be raised in cold weather by heating the mixing water or heating the aggregates, or both. The control is most readily obtained by heating the mixing water. However, the temperature of the water should not exceed 165F because of the danger of causing a flash set of the cement.

Transportation of freshly-mixed concrete in winter presents no particular problem. If it is moved by belt conveyor, the conveyor is housed in a heated enclosure for its entire length. If it is moved by buckets or trucks, no attempt is made to cover the surface of the concrete unless the haul is more than a mile or unless the temperature is well below zero. For longer hauls and lower temperatures, tarpaulins are placed over the concrete.

Where concrete is pumped there is little loss of heat in the pipelines if the concrete is kept moving and, except in the very coldest weather, the pipelines need no protection. However, if pumping has to be stopped on account of delays at the forms, freezing may occur and block the pipes so that protection is advisable for lines that are to be in use over a considerable period. If the pumpcrete line is short, tarpaulins wrapped around the pipe and a steam line laid alongside will provide adequate protection. For longer or more permanent lines, a continuous box is built around the pipe with removable top and sides and a steam line is installed inside. It is important when using pumpcrete in winter to inject live steam into the line before starting to pump. Unless this is done, the first concrete pumped gradually freezes as it progresses along the line.

Forms for receiving concrete in extreme weather must be preheated and the rock surface brought at least above the freezing point. A heating period of several days is common under such circumstances. The air leakage through joints in narrow lumber sheathing make it necessary in cold, windy weather to enclose the form with tarpaulins. When plywood sheathing is used the need for tarpaulins is reduced.

During cold weather, freshly placed concrete is maintained at a temperature not less than 50F for a period of at least 72 hours and protection against freezing is continued throughout the remainder of the 10-day period. In massive structures, the initial heat within the concrete is not readily dissipated as in lighter sections and is added to by heat generated in the hydration of the cement. However, as this chemical heat is not available in large amounts until 24 hours after placing, immediate surface protection is as necessary for massive structures as for others, but less protection is required later.

For massive sections, the top of the formwork is covered and unit heaters are used within the form while placing proceeds. On small sections where the

heat of hydration of the cement is dissipated more rapidly, heat must also be provided between the outside cover and the forms. In all instances special care is taken to protect corners and edges against heat losses. Salamanders and oil stoves are absolutely banned as they have the disadvantages of producing dry heat, emitting fumes and introducing fire hazard. Live steam is particularly advantageous for cold weather protection as it provides moisture as well as heat. The live steam provides another necessary service for cleaning and warming all surfaces where concreting is to start.

Variations in Mass Concreting Practice

For many years the placing of mass concrete by Ontario Hydro has almost invariably been associated with high forms and high lifts calling for closely-spaced tie rods and the use of elephant-trunk drop chutes for placement, especially under overhanging forms. Consequently, the concrete has required a high degree of plasticity and cohesiveness, with the practical maximum size of aggregate being limited to about three inches. In several instances, however, there have been departures from this practice which have apparently resulted in acceptable thermal stress conditions. The most prominent deviation occurred at the St. Lawrence project in placing the walls of the canal diversion structure and the U-abutment at the north end of the powerhouse. There, the contractors elected to use high-tensile form ties at a spacing that permitted bucket placement. Consequently, the lifts were five to seven feet in depth. In spite of the change in lift height, however, thermal conditions approaching those for high lifts were secured by maintaining a covering over the surface during winter conditions until the next lift was placed. This practice prevented undue loss of heat from the horizontal construction joint surfaces and reduced the tendency for cooling shrinkage to take place under conditions of high restraint.

Investigation of Thermal and Structural Behaviour

Program of Investigation

To check the conclusions drawn from theoretical studies and to obtain data on various aspects of the performance of the larger structures, extensive instrumentation was installed in several dams. Included were Barrett Chute, Stewartville, Aguasabon, Pine Portage, George W. Rayner, Des Joachims and Otto Holden. In this program particular attention was paid to the measurement of temperatures and temperature gradients, the occurrence of cracks, and the magnitude and general pattern of joint openings. In addition, core drilling was carried out to explore the possibility of internal cracks in critical locations and extensive pulse velocity surveys were made periodically. The methods of investigation are described under the headings of temperature, strains and joint movement, stress and cracking. Certain other measurements, such as deflection and uplift pressure, were also made but need not be discussed in detail in this paper.

a) Temperature

Resistance thermometers accurate to about 0.1F were installed in the rock foundation at distances up to 20 feet below the base of the dam, near the rock surface, and at various distances from the exposed faces of the structure.

All electrical connections were terminated at measuring stations in the inspection galleries.

b) Strain Measurement and Joint Movement

Throughout these investigations extensive use was made of meters of the unbonded resistance wire type devised by R. W. Carlson,[3,4] to measure strain, shear movement, joint and crack openings. The state of strain, to an accuracy of about four micro-inches per inch, in the vertical plane normal to the axis of the dam was determined at each installation point by measurements in three directions at 45 degrees to each other in this plane. One meter, placed at right angles to this plane, measured strain parallel to the axis of the dam. In addition, stress-free concrete cylinders containing strain meters were placed in cavities formed in the interior of the dam near each group of embedded strain meters. The difference between the length changes of a "no stress" strain meter and that of the nearby embedded strain meters is then considered to be the change due to load.

Joint meters were installed in the dams to measure the extent to which the vertical construction joints between sections opened as a result of thermal contraction, creep and any time-dependent departures from dimensional stability that the concrete might exhibit. Similar meters were also installed to measure the shearing movements, in a direction normal to the longitudinal axis at the vertical joints and at the interface between the base of the dam and the rock foundations.

c) Stress Measurement

In two dams, a limited installation of Carlson meters was made to explore the feasibility of direct measurement of stress.

d) Crack Detection and Exploration

Crack meters were embedded in the mass concrete to detect major vertical cracks, parallel to the longitudinal axis, in the interior of the dam. These meters consisted of a special Carlson strain unit, attached to a 10-foot long, 5/8-inch diameter, steel rod. A greased cotton sleeve, pulled over the entire length of the rod and meter, prevented bonding of the concrete except at the extreme ends. These elongated meters were placed in a horizontal line in the concrete perpendicular to the longitudinal axis of the dam; their ends were overlapped 2 inches to avoid any gaps in which cracks could remain undetected. In one dam eight rows of these meters were installed at different locations and elevations.

The search for internal cracks was pursued further using core drilling and pulse transmission techniques. In Barrett Chute Dam holes inclined slightly above the horizontal were drilled from convenient locations above the toe backfill towards the upstream face in order to intersect any vertical cracks that might extend a significant distance above the foundation. It was felt that the inclination of these holes would cause a core to separate at a vertical crack and slip back into the core barrel, thereby limiting damage of the crack surfaces, which could then be inspected. At Stewartville Dam, where the base thickness was too great to permit successful transmission of ultrasonic pulses between the downstream face and the inspection tunnel, vertical holes were drilled to permit two intermediate transducer positions. Special transducers capable of being lowered into the 6-inch diameter drill holes allowed the base to be surveyed for internal flaws for the entire distance

between upstream and downstream faces.

Surface cracking was investigated extensively in a number of structures. The dye injection and drilling technique was used initially to ascertain the maximum depths of such cracks and the trend in depth over a period of years.(5) Following the development of the Soniscope, both surface and internal crack surveys were facilitated greatly.(6)

Observations and Analysis

a) Temperature Studies

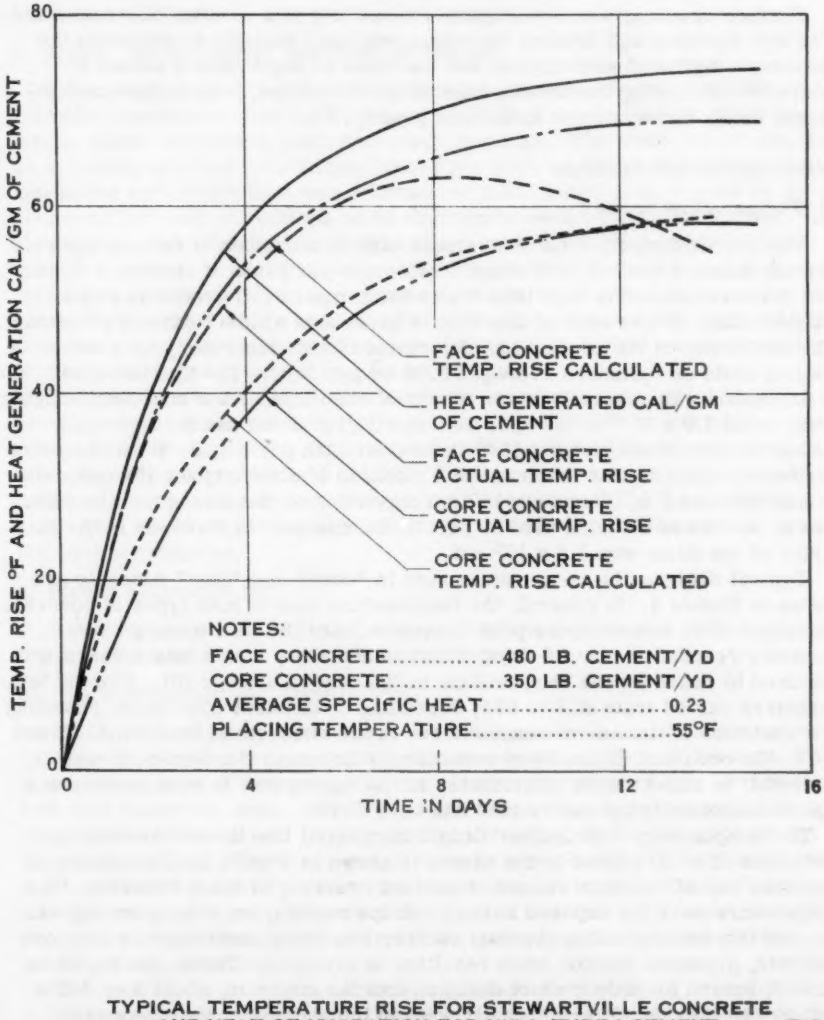
The heat of hydration for the cement used in a number of Ontario Hydro's gravity dams, Figure 3, was about 80 calories per gram of cement at 28 days. For concrete placed in high lifts where the temperature generally rises adiabatically, 85 per cent of this heat is generated within 7 days of placement. The diffusivity of the concrete as determined in the laboratory by a series of cooling tests on cylinders averaged 0.04 ft^2 per hour. The thermal coefficient of expansion was generally lower than published figures for concrete, ranging from about 3.0×10^{-6} inches per inch per degree F for the St. Lawrence Power Project to about 4.5×10^{-6} inches per inch per degree F for the more northern projects. The instantaneous modulus of elasticity for the concrete varied between 5×10^6 and 6×10^6 psi depending on the strength. The effective or sustained modulus used in part of the analysis of stresses in the interior of the dams was 2.5×10^6 psi.

Typical data for the temperature rise in "core" and "face" concrete are shown in Figure 3. In general, the temperature rise of both types of concrete was about 60F; however, the peak temperature of the face concrete was normally reached in about 9 days, whereas about one month was frequently required to reach a peak temperature in the core of a large lift. Placing temperatures varied from 45F to 60F, depending on ambient conditions, resulting in a variation of maximum temperatures in the concrete of between 115F and 130F. On occasion, it has been necessary to increase the cement content somewhat to offset minor deficiencies in the aggregate. In such instances a typical adiabatic temperature rise has been 75F.

The temperature distribution along a horizontal line drawn through the centerline of a lift placed in the winter is shown in Figure 4. This figure illustrates one of the main causes of surface cracking in mass concrete. The temperature near the exposed surfaces drops rapidly, especially during winter, and the corresponding thermal contraction, being restrained by the core concrete, produces tension often resulting in cracking. These cracks, when formed, extend for only a short distance into the concrete, about 2 or 3 feet and, though they do not significantly weaken the structure, they may contribute to surface deterioration and to possible leakage at vertical joints.

Figure 5 shows the temperature history of several points in the critical region near the base of a dam. The temperature gradients are much less than at an exposed surface, hence helping to prevent cracks. The rise and subsequent fall in rock temperature is also a favourable factor in reducing concrete stress.

By the time the core concrete reaches its peak temperature there exists essentially the condition of simple cooling with superimposed ambient temperatures at the surface. The time required for the temperature at the centre of Stewartville Dam to reduce to within 5F of its final value of 50F was about 5 years. The annual temperature variation in the mass concrete of a

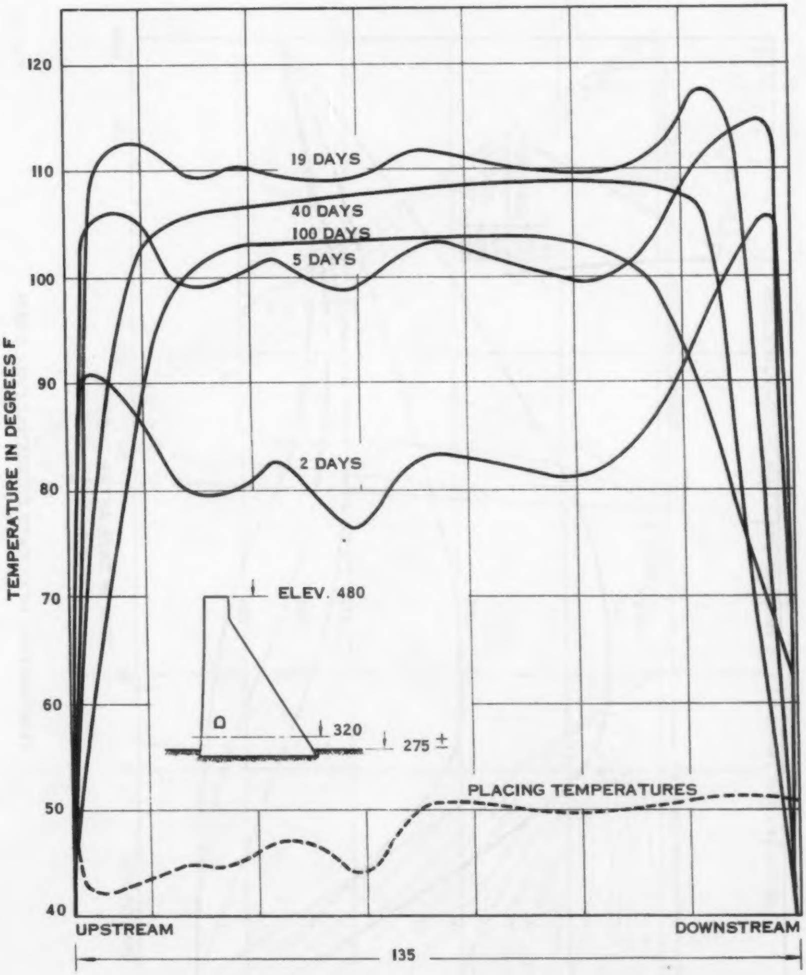


TYPICAL TEMPERATURE RISE FOR STEWARTVILLE CONCRETE AND HEAT OF HYDRATION FOR HULL TYPE I CEMENT FIG. 3

typical Ontario Hydro dam is shown in Figure 6. The temperature in the central part of the lower half of this structure does not vary by more than two degrees F due to ambient changes.

b) Thermal Stresses

Two types of thermal stress were the object of detailed study; - differential stresses developed during the cooling down period from restraint to contraction presented by the internal mass, and those developed due to restraint to contraction presented by the foundation rock. An elementary method of



TEMPERATURE PLOT – STEWARTVILLE DAM
STATION 4+04 – ELEVATION 320.0

FIG. 4

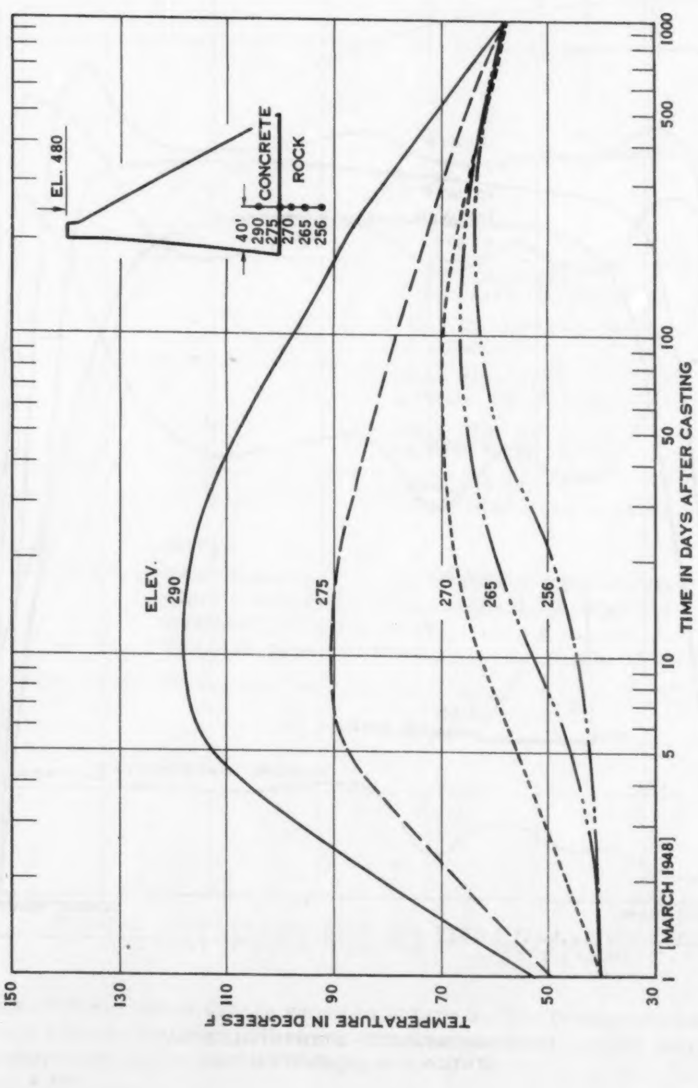
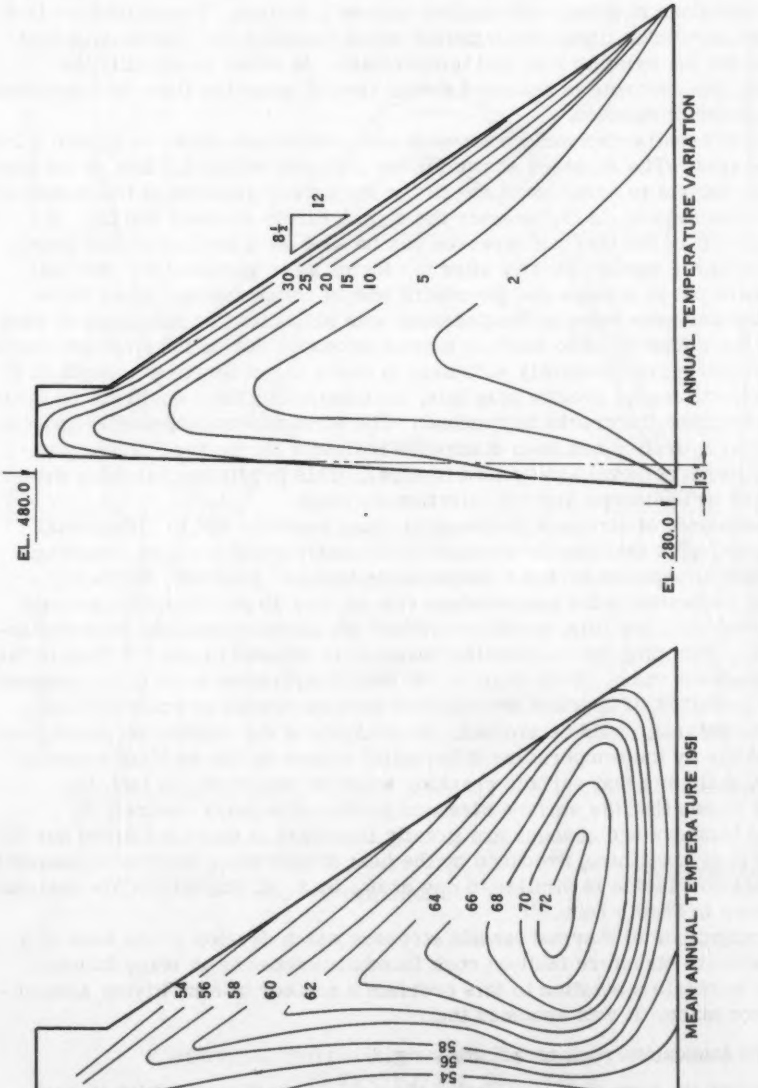


FIG. 5

TEMPERATURE NEAR BASE OF STEWARTVILLE DAM



TEMPERATURE THREE YEARS AFTER COMPLETION OF CONSTRUCTION
STEWARTVILLE DAM – BLOCK 12 – STATION 4+44

FIG. 6

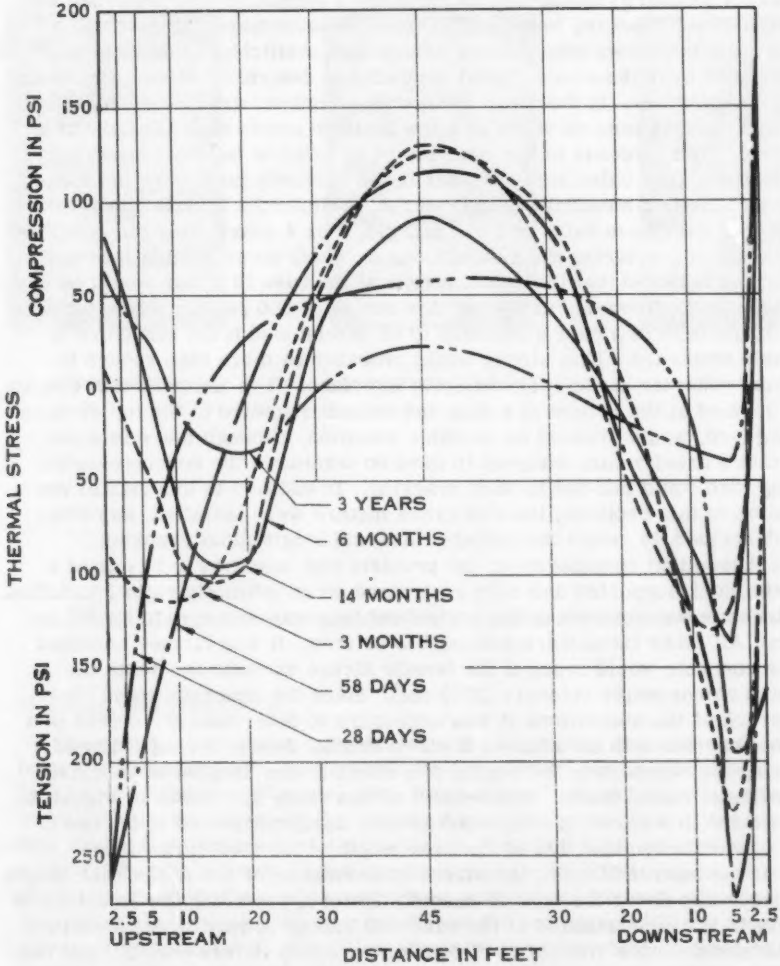
computing the differential thermal stresses was first derived by R. W. Carlson and has since been used to analyze thermal stresses in a number of structures.(7,8,9) In this method it is assumed that a section which is plane before the temperature change remains plane. This means that there is a linear variation in actual deformation across a section. The procedure is to find that particular linear deformation which satisfies the condition of equilibrium for the existing load and temperature. In order to simplify the analysis, the concrete is assumed stress free at about the time the maximum temperature is reached.

The differential thermal stresses in one section are shown in Figure 7 for various ages. The stresses shown for the concrete within 2.5 feet of the surface are subject to some doubt due to the very steep gradient of the temperature in this region. In the interior the stress rarely exceeds 100 psi. A distinct shift of the thermal stresses can be seen as a section cooled over a period of three years. Shortly after the forms were removed the external faces were put in tension and the centre was in compression. After three years the surfaces were in compression with slight tension indicated on each side of the midpoint. The surface tension produced during the first few weeks after placement is frequently sufficient to crack the concrete to a depth of 2 or 3 feet. At depths greater than this, the tensile stresses would not be large enough to cause the cracks to continue. The stress distribution after most of the heat of hydration has been dissipated indicates the surface cracks produced at form removal should tend to close. This prediction has been substantiated by Soniscope and dye injection surveys.

The analysis of stresses produced in mass concrete due to differential cooling indicates that tensile stresses sufficiently great to cause cracking, occur only at exposed surfaces where large thermal gradients frequently exist. A reduction in the temperature rise of, say 30 per cent, which could be obtained with low lifts, would not reduce the surface cracking tendency appreciably, providing the surface temperature is allowed to quickly drop to the winter ambient value. Even if all of the heat of hydration were to be removed as it is produced, it appears doubtful that surface cracks in mass concrete, placed in Ontario, could be avoided. An analysis of the surface stresses produced solely by the temperature differential caused by the ambient seasonal changes, indicates that surface cracking would be expected. In fact, the analysis shows that the surface stresses produced in mass concrete by seasonal temperature changes are greatly in excess of those produced due to the rise in temperature, produced by the heat of hydration, and the subsequent fall. This conclusion is similar to one drawn by J. M. Raphael in his analysis of stresses in Shasta Dam.(9)

The magnitude of thermal tensile stresses which develop at the base of a mass concrete structure built on rock foundation depends on many factors. In order to obtain a solution to this problem a number of simplifying assumptions were made. It was assumed that:

1. The foundation rock is infinitely rigid.
2. During the heat evolution period the creep is sufficiently high that no significant thermal or other stress remains at the time maximum temperature is reached. During cooling, creep has so decreased that the concrete can be considered rigidly bonded to the rock.

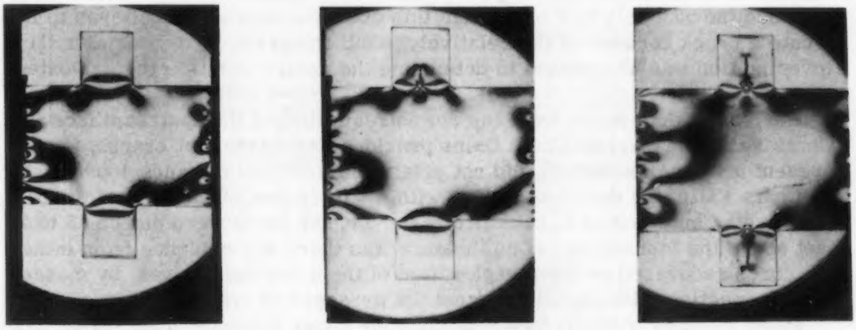


VERTICAL THERMAL STRESS – STEWARTVILLE DAM
ELEVATION 365, SECTION II

FIG. 7

Two slightly different methods of calculating these stresses were used. In one case a measured instantaneous modulus of elasticity was combined with a "creep-time" function, based on previous measurements, to determine stress from the known temperature change and coefficient of expansion.[10,11] Calculations were done on a digital computer to determine stress across the base of a structure. In the other method an effective modulus of elasticity was used to determine stresses at a few isolated points near the base of a structure. The stresses in the concrete were found to be about equal for each method. The calculated stresses in the concrete just above the foundation rock based on measured temperatures, indicate the tensile stresses increase to a maximum value of about 500 psi, 3 or 4 years after the concrete was placed. Considering the assumptions on which these calculations are based, it is believed that the actual stress at the base of a dam would be considerably less. However, accepting this stress of 500 psi tension in the concrete at the base of a dam it remains to be determined if the structure is seriously weakened. This stress would probably be more than enough to crack the concrete at the rock-concrete interface. It is assumed that cracks which formed at the bottom of a dam and extended upward to the top or to an exposed face, would produce an unstable situation, although there is some doubt that a gravity dam designed to have no tension in the concrete under working load could fail due to such cracking. In addition to theoretical consideration of this problem, lines of crack meters were installed, as previously described, to detect the extent of internal longitudinal cracking.

In a theoretical consideration, the problem was assumed to be one of a concrete block supported and fully restrained on an infinitely rigid foundation. The stress in the concrete at the restrained face was assumed to be 500 psi tension. All other faces were assumed to be free. It was further assumed that the concrete would crack if the tensile stress exceeds one-tenth the minimum compressive strength (2000 psi). Since the concrete would obviously crack at the foundations it was necessary to determine the stress in a block of concrete with an unknown depth of crack. Due to the difficulty of evaluating the stresses in the region of a crack it was decided to undertake a photoelastic model study. The results of this study are shown in Figure 8. The stresses in a model of uncracked gravity dam, restrained at the base, extend upward only about 1/8 of the base length of the structure. If the stress at the base is 500 psi, the stress at a distance of 1/8 of the base length of the structure above the base is essentially zero, providing the structure is uncracked. A crack initiated at the base will extend upward in the structure until the stress in the vicinity of the end of the crack is less than 200 psi tension. Saw cuts made in the models to simulate cracks indicate that cracks have little influence on the stress pattern. If the cracks extend only a short distance above the base, the stress will be concentrated at each end of the crack. However, if the cracks extend a distance equal to about one-fifth the base length of the structure the stress in the region of the end of the crack is reduced to about one-sixth of that at the base, i.e., 80 psi. Assuming a stress of 500 psi at the base of a gravity dam it is concluded that cracking would not extend beyond a distance equal to about one-eighth the base length of the structure. It would appear from this study that vertical cracks in a structure can be controlled by simply placing the concrete so that it cools as a monolith, even if the temperature rise is high in contrast with that typical of low-lift practice.



(A) UNCRACKED DAM (B) WITH SHORT CRACK AT BASE (C) WITH LONG CRACK

STRESS PATTERN IN PHOTOELASTIC MODELS OF DAM RESTING ON ROCK FOUNDATION FIG. 8

c) Strain Measurements

The principal stresses near the base of two gravity dams were determined from the measured strain corrected by "no-stress" strain observations and by applying "Poisson's" ratio. The "load-strain" multiplied by an effective modulus gave the stress in the region of measurement. Before the structure was loaded the maximum principal stress occurred near the upstream face. After the head pond was raised the maximum principal stress occurred near the midpoint of the base of the structure accompanied by a slight change in direction. The maximum principal stress did not exceed 500 psi at any of the points of measurement. No observations were made which suggest excessive loading, although the methods used to arrive at these loads are known to be inaccurate. Several authors have criticized this method of calculation and have estimated that the error could be as much as 100 per cent.(12,13) Even if an error as large as 100 per cent exists in the results the load stresses would not be excessive. Attempts that were made to determine stresses directly using stress meters were not successful.

d) Detection of Internal Cracks

In general, the lines of crack meters embedded in Stewartville Dam indicated there were no longitudinal cracks in the structure at an elevation 20 feet above the foundation rock. No meters were installed closer to the foundation than 20 feet so that no information was provided on cracking adjacent to the rock. In only one case was an irregular extension observed in the readings

of a crack meter, which may have indicated cracking in the concrete. This meter indicated an irregularity six months after placement when the temperature decline was only 20F. Although this observation was not believed to indicate a crack because of the relatively small temperature decline a further investigation was undertaken to determine the nature of this irregular extension.

Horizontal drill holes from the downstream toward the upstream face in Chats Falls and Barrett Chute Dams provided assurance that cracks, if present near the foundation, did not extend a significant distance above it. At Chats Falls, the drill holes were within 8 to 10 feet of the base of the dam; at Barrett Chute, owing to back fill on the toe, the holes were placed 15 to 20 feet above the foundation. In no instance was there any evidence from either the cores recovered or from exploration of the holes themselves, by means of an inspection telescope, to suggest the presence of cracks.

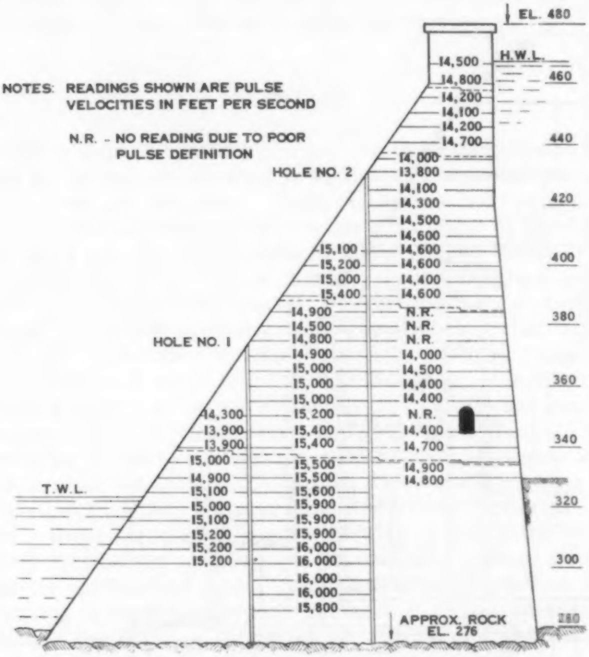
Pulse velocity measurements through the upper portions of all major dams failed to detect the presence of internal cracks within the limited range of the Soniscope. Vertical holes, drilled in one section of the Stewartville Dam, permitted essentially complete coverage of the concrete in this section, Figure 9. Although the survey disclosed no cracks in the central portion of the base, it directed attention to a relatively small zone, below one of the construction joints, in which abnormal measurements were obtained. The pulse transmissions could not define the exact cause of this abnormality but since the block was placed during the winter and two months elapsed before concreting of the next lift commenced, superficial thermal cracks may have formed at the top of the block. In any event, the abnormality is confined to the upper part of one lift and is not considered to constitute a hazard to the structural stability of the dam.

e) Surface Crack Surveys

Records of crack depths in the major structures showed that the most significant cracks occurred in the access tunnels of the dams. Due to their inaccessible location upon completion of the structure, it was not possible to follow their behaviour over a period of years. However, there is evidence that they did not extend beyond the lift in which they originated. Cracks which appeared on the downstream face shortly after removal of the forms had initial average depths between 2 and 3 feet, but within a short time they decreased appreciably in depth and width, eventually approaching an average of 4 to 6 inches. Some of the pulse survey data obtained at the G. W. Rayner Dam are shown in Figure 10 to indicate the nature of the crack pattern.

f) Joint Movements

Openings of vertical construction joints at Stewartville Dam are believed to be typical for a gravity dam built in large lifts with no artificial cooling. In general, the joints begin to open shortly after the concrete has reached its maximum temperature, an indication that the concrete is relatively plastic during the temperature rise. The joints open first near the exposed surfaces where the heat is dissipated at a relatively high rate. The openings at the centre increase with the decrease in internal temperature and approach a fixed value of about 0.075 of an inch, (for sections 40 feet in width), upon dissipation of all the heat of hydration. Annual variations in joint openings, about equal in magnitude to those caused by the loss of heat of hydration,



PULSE VELOCITY MEASUREMENTS — STEWARTVILLE BORE-HOLES FIG. 9

occur near the downstream face. These changes were found to bear a close relationship to the annual temperature variation in the concrete.

The observation of shearing movement between adjacent sections of a dam indicate little or no relative movement between adjacent sections which are of equal height. However, for adjacent sections which are greatly different in height, significant shearing movements occur due to water and thermal loads. A high section adjacent to a shallow section results in a transfer of part of the load on the high section to the latter, provided the sections are keyed.

Shearing movement at the junction between the foundation rock and the concrete was found to be negligible.

g) Horizontal Deflections

In each dam where internal plumblines were installed the horizontal deflection due to water load may be accounted for if the dam is assumed to be free from large cracks and is constructed of good quality concrete resting on a rigid foundation. In each of the structures measured there appears to be a slight "creep" of the top of the structure downstream with respect to the bottom, over a period of years.

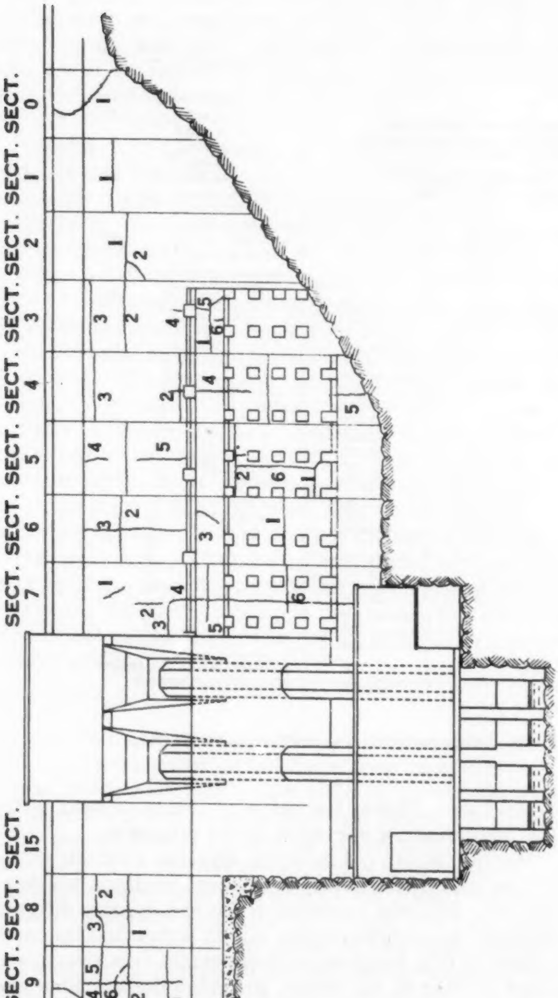


FIG. 10

ULTRASONIC INVESTIGATION OF SURFACE CRACKS
G. W. RAYNER GENERATING STATION

h) Uplift Pressures

Uplift data, collected over the past ten years on six dams, show no abnormalities attributable to unsoundness of these structures.

SUMMARY

The high-lift method of construction as used by Ontario Hydro has been characterized by lift-heights ranging up to 50 feet and occasionally more as practical considerations permitted. The practice has not required temperature-control measures except for the prevention of unduly rapid cooling of lift surfaces and of formed surfaces under severe winter conditions. It appears that all structures built to date are demonstrating a complete absence of structurally dangerous cracks. Present evidence indicates that the absence of serious internal cracking is the result of the relatively free thermal expansion and contraction of entire monoliths in which foundation restraint is restricted to a small fraction of the height of monolith or lift.

Although the system has proven eminently suitable for the structures described, especially when construction had to be carried right through the winter months, the cement requirement, limitation on aggregate size and costs of forming may weigh strongly against its adoption in other situations. Recent experience suggests, however, that for the sizes of structures built by Ontario Hydro, these objections may conceivably be overcome by approaching, with low-lift construction, the high-lift principle of continuous placing. It is felt that the condition of moderate horizontal restraint may be achieved if serious cooling of construction joint surfaces is prevented by either thermal protection of the surfaces or by placing a suitable upper limit on the time interval between lifts.

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Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

BOX CANYON HYDROELECTRIC PROJECT[a]

Closure by Arthur P. Geuss

ARTHUR P. GEUSS.[1]—The author appreciates the opportunity to amplify his paper as suggested by Mr. F. L. Lawton.

During the hydraulic model tests, with symmetrical piers, high unbalanced forces were apparent by observation. Piezometers were then installed on both faces of the left hand intermediate pier (looking downstream). The unbalanced pressures were found to increase with the discharge and reached a maximum of 24 feet (350,000 cfs), with considerable surging. Conditions were worse at this pier and caused by the general configuration of the canyon upstream from the spillway. The maximum measured differential pressure with the asymmetrical shaped pier was sixteen feet. With a flow of 170,000 cfs, this differential pressure drops to nine feet. Surging was also reduced to a great extent.

The air vents were designed using model test data developed by the TVA in connection with some of the early TVA projects.

The sand backfill was end-dumped into water with the surface built up five or six feet above the water table. This provides a firm working surface and surplus water from the jetting and vibration operations can be easily drained aside. Very little compaction is obtained in this top layer above the water table. The primary consolidation or compaction takes place below the water table. Some additional increase in the average relative density would have been obtained with a closer spacing in inserting the vibroflotation unit; however, the 70 per cent minimum relative density obtained was believed to be the maximum necessary.

The steel sheet piling was fitted to the rock abutments to the maximum practical extent. A grout pipe was then jetted down into the adjacent sand foundation and grout pumped in as the pipe was withdrawn. This operation was repeated several times on both the upstream and downstream sides of the sheeting to insure adequate mixing of the grout and sand. In addition, holes were drilled and grouted in the rock adjacent to this junction in order to close any fissures or joints that would permit seepage to bypass the steel sheet pile cutoff.

Only horizontal earthquake forces were considered in the design of the spillway.

The head loss in the forebay channel in the prototype ranges from 0.15 feet with a flow of 10,500 cfs to 0.80 feet with a flow of 29,000 cfs. The head loss for a given flow will, of course, vary with the headwater elevation. There

a. Proc. Paper 1672, June, 1958, by Arthur P. Geuss.

1. Vice-Pres. and Chf. Engr., Harza Eng. Co., Chicago, Ill.

were no marked irregularities in the flow distribution in the forebay channel and to each unit either in the model or the prototype.

UNDERGROUND POWER PLANTS IN SCOTLAND[a]

Corrections

CORRECTIONS.—In the February 1959 Journal of the Power Division, on page 150, the discussion by P. L. Aitken is, in fact, a discussion of Proceedings Paper 1675 by C. M. Roberts and should, therefore, have appeared on or about page 144.

a. Proc. Paper 1675, June, 1958, by C. M. Roberts.

ROCKFILL DAMS: PERFORMANCE AND MAINTENANCE OF DIX RIVER DAM[a]

Closure by L. A. Schmidt, Jr.

L. A. SCHMIDT, Jr.,[1] M. ASCE.—Mr. Lawton in his discussion requests comment on whether the two large east cliff shots contributed to leakage at the surfaces on which the rock resulting from these movements was deposited. It is not likely that this is the case because practically all of the overburden in the deposited areas had been removed and the rock from the shots above fell or moved down with substantial impact that must have had some settling influence. It can be stated, however, that the sluicing operations were not as effective as they should have been for smaller lifts and therefore were much less effective for the greater volumes deposited momentarily.

Parenthetically, the large shots are believed to have had a much greater detrimental effect by virtue of the concussions created, which loosened blocky rock formations and opened potential paths for sidehill leakage. This effect was noticeable particularly in the tunnel after the blasts, where considerably more scaling was required, particularly in the crown of the arch than would have been necessary had the blasts not been pulled. This is known because the tunnel had been dug when the first blast was made but had not yet been concrete lined.

The physical properties of the limestone rock at Dix are as follows: It is a dense, compact finely crystaline, light gray rock. Its unconfined crushing strength would be in the range of 10,000 to 20,000 pounds per square inch. The small portion of silt and clay impurities forms a coating on the rock after long exposure giving it a characteristic drab gray color. There has been no apparent deteriation of the rock in the visible portions of the fill, nor in the abutting cliffs.

Unfortunately actual maintenance costs for this structure are not available. Except for the drilling and grouting and some minor patching of the concrete face, maintenance as such has required only minimum expenditures. However the tunnel repair occasioned a substantial expenditure all at one time when failure occurred. It is possible that this work could have been spread out over several years of maintenance and the obvious ultimate failure averted. However the need for electric energy during and immediately following the war years practically precluded such maintenance work because it would have meant shutting the plant down while the work was being done and every possible kilowatt of energy was needed for production until other planned generating could be constructed and placed in operation.

a. Proc. Paper 1683, June, 1958, by L. A. Schmidt, Jr.

1. Pres., Schmidt Eng. Co., Inc., Chattanooga, Tenn.

DESIGN AND CONSTRUCTION OF THE AMBUKLAO ROCK FILL DAM[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The authors have made a notable contribution to the design and construction of rockfill dams with their treatment of the 430 foot high central core Ambuklao Dam.

Having visited the site during the period of initial investigations, the significance of the topographic conditions and, to a somewhat lesser degree, the geologic features was apparent. Thus the authors' observations on the site geology are welcomed as a penetrating statement of the distinctive differences between the geologic conditions quite commonly found at damsites in the tropics and in temperate regions. Justification for the paragraph "The geologic conditions at damsites in the tropics - - are more familiar" is abundantly afforded by Ambuklao geologic features as well as those at many other tropical sites.

The authors have taken full advantage of the topographic situation in utilizing the additional head of about 60 feet in connection with diversion through tunnel "C", and perhaps more significantly in utilizing 180 feet additional gross head (with respect to the normal 505 feet) made possible by the 7200 feet tail tunnel. Could this head have been developed any other way at anything like the cost achieved?

It is interesting to note that "Based on the evidence from these borings and surface observations, it was concluded that the rock in the abutments and in the river bed was not sufficiently strong to withstand the pressure which would be developed by any economically feasible type of concrete dam". Foundation conditions and availability of economically-utilizable core and shell materials are the two principal reasons rock and earthfill dams are likely to be more and more widely used in relatively undeveloped tropical countries as well as elsewhere.

The authors observe that "Except for the foundation of the core, the existing river gravel was left in place". Could they state the approximate maximum thickness?

The authors' crushing tests on the samples of broken rock from the quarry areas are particularly interesting, having demonstrated the diorite was subject to greater breakdown than the metamorphics during blasting, handling and placing in position, contrary to expectation from visual inspection. This demonstrates the value of adequate tests on all components of a rock fill dam.

Although no reference thereto is apparent from the text, could the authors advise if any attempt was made to determine the pore pressures in the three field test fills?

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a. Proc. Paper 1864, December, 1958, by E. Montford Fucik and Robert F. Edbrooke.

1. Chf. Engr., Power Dept., Aluminium Laboratories, Ltd., Montreal, Canada.

The design studies which led to the use of a rockfill with vertical core rather than a sloping one demonstrate clearly conditions which may sometimes be overlooked; i.e. length of seepage path due to increased contact area between the core and the foundation, and increased loading on the contact area with probable minimization of leakage at the contact. Could the authors state the relative weight given these two factors as compared with the saving in fill materials?

The vertical settlement and downstream deflection have been most moderate, at less than 6 inches and about 3 inches respectively. How does the vertical settlement compare with that predicted from tests such as shown by Fig. 7?

It is to be hoped National Power Corporation engineers will, in due course, make available to the profession continuing observations on performance of the outstanding Ambuklao rockfill dam and particularly on vertical settlement and downstream deflection.

HYDRAULICS OF CIRCULATING SYSTEMS[a]

Discussion by R. T. Richards

R. T. RICHARDS.[1]—Mr. Bolieau has presented a valuable review of the basic elements which influence the design of a typical circulating water system. Technical literature is sparse concerning this very important element of steam power plant design. Much of the information presented in this paper can be readily applied to circulating water systems in general as well as to the TVA plants. It is apparent that the TVA C. W. Systems are quite similar, however, and do not present the range of problems encountered in widely variable site situations.

The writer would like to comment on a few of the aspects of circulating water system design which Mr. Bolieau has covered.

Valves and Flow Control

Mr. Bolieau notes that the TVA has gone from costly cone valves to butterfly valves for pump discharge control. The present TVA valves seem to be quite complex in operation, however, and utilize 2 speed control for opening and closing. Such refinement is rarely necessary. The writer's firm has made a careful study of the controls required for safe and efficient operation at a large number of stations with varying pump and site characteristics. This study has resulted in the evolution of a simplified control system, the elements of which are as follows:

1) A single speed motor operated butterfly valve with opening and closing times set at 20 seconds for any system less than 3000 feet in length.
2) On pump start-up, valve and pump are energized together and the valve moves to full open position in 20 seconds. This is fully satisfactory for all but axial flow pumps where the shutoff horsepower requirements may be very high. In this relatively rare case controls may be designed to open the valve about 15% before pump is energized.
3) On pump shutdown the tripping of the pump and the start of the 20 second valve closing cycle are simultaneous.
4) Following power failure to the pump the valve will close in 20 seconds in the same manner as for normal shutdown. Mr. Bolieau points out that following interruption of pumping due to power failure the TVA valves are closed by manual control after the system has come to rest. Surge analysis and field tests by the writer do not indicate that this precaution is necessary. Valves should be closed, however, before the pumps are started up again.

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a. Proc. Paper 1946, February, 1959, by Clifton W. Bolieau.
1. Hydr. Engr., Ebasco Services Inc., N. Y. C., N. Y.

For systems over 3000 feet in length the valve timing must be reviewed. Longer times may be advisable to limit surges. Also, if for some reason the operators require a control that will permit closing the valve before stopping the pump a considerably longer valve closure time will be required to prevent severe water hammer. The longer total valve closure time will be necessary to give a reasonable net time for actual stopping of the water flow. It can be seen that as a valve closes against an operating pump the head upstream of the valve rises; the valve will be well closed before a significant and rather sudden flow cut-off takes place. Negative surges downstream of the valve may be severe enough to result in water column separation and subsequent severe return wave shock.(1)

It can be seen from the foregoing discussion that the controls discussed are less costly and complex than two speed valve motors and the associated controls described by the author. These simplified controls have been successfully used in a number of both large and small capacity circulating water systems.

Filling an Empty System

For filling an empty system the writer recommends manual pushbutton control of the valve. The valve should be stopped at a 15% open position and the system filled before the valve is opened fully. Although it is not conservative practice to fill the circulating water system with valve wide open, it is frequently done on systems less than say 1000 feet in length. As Mr. Bolieau pointed out, there are systems with pumps close to the condenser and no valves at all. The writer's firm has put valveless systems in operation where pumps are 200 to 400 feet away from the condenser. Of course, each pump operates on a completely separate water circuit. Very careful observations of startups over several years have indicated satisfactory operation free from objectionable surges.

Axial flow pumps present a special problem where low flow brake horsepower may be too high for the motor. For system filling the condenser must be primed before pump startup in order to establish the normal operating siphon and limit the pump operation to its normal range. The priming method mentioned by Mr. Bolieau would be applicable in this case.

Operating Instructions

Mr. Bolieau has noted that complete operating instructions are provided by the TVA designer for the operator's use. This is a very important matter. The writer is aware of cases where a designer has provided a carefully thought out water system designed for many operating conditions and possibly quite complex, then failed to provide proper instructions for its operation. Particular care should be taken to insure that the operators and plant mechanical and electrical engineers understand the hydraulic problems involved in operating the large and costly pumping stations entrusted to their care.

Flow Throttling

In the matter of operation, Mr. Bolieau has noted that a throttling valve is sometimes necessary to keep the pumps from operating too far out on their characteristic curves and thereby subjecting the pumps to undue wear. His Fig. 3 illustrates such a case. The writer has three comments on this point.

First, the necessity for throttling should be reduced as far as possible by the correct choice of pumps capable of operating over as wide a range of head as may be necessary. Secondly, if the conditions are so extreme that throttling will still be required the valves must be specifically designed for this service; an operator cannot use just any convenient butterfly valve for flow throttling. Lastly, the operating conditions which will require throttling must be carefully set forth in the operating instructions. It may not be evident to an operator that a pump is operating in an undesirable range until cavitation pitting or undue bearing wear has sharply reduced the life of the pump.

Air Binding

Mr. Bolieau has presented a very useful discussion on the presence and removal of air in the system. He has limited his discussion to air release at the condenser and occasional air binding at the pump discharge. It has been the writer's experience that air is frequently removed from the condenser by the natural pumping action of the water in the piping at the condenser discharge; air removal equipment is thus not necessary and can be omitted from many systems. Whether the condenser will be self venting appears to be quite unpredictible. Provision for the future installation of vacuum pumps should always be made. There is no question at all, however, that air binding will occur in conduits sloping downhill, especially if those conduits are under vacuum.(2) Mr. Bolieau has made a valuable addition to the design of air evacuation systems both by presenting a simplified chart of the amount of air in water and by noting the TVA adoption of air removal equipment to handle 10% of the theoretical air release. The writer has verified this 10% figure on a 66", 84" and 96" siphons running over river levees in southern power plants. The use of Mr. Bolieau's chart neglecting temperature effects except at atmospheric pressure appears to be justified when vacuums are high; the effect of temperature rise on air release is very small since the low pressure itself will result in the theoretical release of most of the air. In systems with less severe vacuums the writer prefers to use Van Hengel's charts to which Mr. Bolieau referred in his Ref. 2. These charts provide an accurate theoretical volume of the vapor mixture lying above the water surface in the conduit. For several cases the writer has checked, Van Hengel's charts give as much as 25% difference in total theoretical air volume compared with results accounting for pressure only. As the author has noted, however, only a small proportion of this air must be removed from the system by artificial means. The rest remains in solution or in small bubbles and is carried out of the conduit.

The author's paper shows an upward sloping condenser discharge tunnel in Fig. 2. This will certainly aid in getting air bubbles out of the system. Level conduits will probably be fully satisfactory also. Downward sloping conduits, however, may lead to a serious air binding problem as noted above. An inexperienced designer may try to follow the ground slope down to the discharge point, under the mistaken impression that he is minimizing excavation and providing drainage without sacrificing efficiency. He may find instead that he has provided a pipe that will not flow full and where head loss will consequently be excessive.

Siphons

Mr. Bolieau has noted that operating vacuums considerably in excess of those at TVA stations have been used. The writer can confirm this statement

from his firm's experience on many siphon designs for recent stations. Some have been designed to operate near 30' vacuum and most are designed to operate at 28' vacuum, related to sea level plants. No difficulty has been experienced maintaining these vacuums, although required air evacuation equipment becomes comparatively large in capacity at these high vacuums.

REFERENCES

1. "Water Column Separation in Pump Discharge Lines" by R. T. Richards. Trans ASME Aug. 1956, Vol. 78, No. 6.

2. "Air Binding in Large Pipelines Flowing Under Vacuum" by R. T. Richards. ASCE Proceedings Hydraulic Division Paper 1454, Dec. 1957.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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c. Discussion of several papers, grouped by divisions.

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